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NUMERICAL MODELS FOR SIMULATION OF THE SEISMIC BEHAVIOUR OF RC STRUCTURES – A CASE STUDY

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Abstract. Seismic analysis of RC structures requires realistic but simple analytical models. In this paper are presented two models proposed and implemented in the program PORANL, to represent the RC buildings behaviour under earthquake loading. The first model represents the bending behaviour of slender RC elements, and the second model represents the influence of infill walls in the global building response. The models are calibrated with test results on full-scale specimens.

It is studied the vulnerability of one building, representative of the Modern Architecture, in Lisbon. Non-destructive tests on materials and natural frequencies measurement were performed to calibrate the model. The building was studied, with the proposed non-linear models, and the seismic safety was evaluated according to recently proposed assessment procedures. Additionally, it is proposed and studied a retrofitting solution, to improve the seismic performance of the building, based on a bracing system with a damping device associated.

Keywords: Non-linear analysis, RC buildings, seismic analysis, vulnerability, seismic retrofitting.

1 INTRODUCTION

The study of the seismic vulnerability of existent buildings in urban areas with moderated/high seismic risk is of extreme importance, to evaluate its safety according to the recently proposed international codes and recommendations. The high number of buildings constructed in Lisbon, Portugal, in the fifties, with particular modern architecture style and characteristics reveal a deficient seismic behaviour. In this work it was studied the seismic vulnerability of an existing building representative of the modern architecture style. The 3rd parcel from the Unity Type A in the Infante Santo Avenue, in Lisbon, is a singular example of Modern Housing Project in Portugal [1]. It's a building inserted in an Urban Plan initialized by Alberto José Pessoa (1919-1985), in 1947, when he joined the "Urbanization Study on the Protection Area of Palácio das Necessidades" in Câmara Municipal of Lisbon (CML). It consisted in dividing and expropriating land parcels for the new Infante Santo Avenue, which completed the ring created by De Gröer's Municipal Plan (1938-1948).

Between 1949 and 1951, the CML and A. Pessoa began the urbanization area process with "Beforehand Project for the residential and commercial Complex in Infante Santo Avenue Central Area", which included

31 parcels along the new Avenue. The transverse section of the proposal reveals a volumetric composition that recreates the landscape, showing an obviously modern matrix influence. The buildings look like free objects claiming that "architecture is an awareness game, with correct and magnificent volumes united under the light" [3], with rectilinear surfaces where it can be seen that "the generators revealing simple forms" [3] and with a plan order that expresses a primary determinated rhythm "with consequences extended from the simplest to the most complex, respecting the same law. A unity law is the law of a good plan: simple law and infinitely modulated" [3].

In 1954, A. Pessoa assumes a "Construction Project of the Infante Santo Avenue between the old Aqueduct and Santana à Lapa Street" and he participates, in CML, in the 1st Work Team with the architects Hernâni Gandra and João Abel Manta, and with Jordão Vieira Dias, the engineer responsible for the structural project.

2 UNITY TYPE A

This team has done the project of the Unity Type A, which are composed by one housing block and another one to the commercial shops. These models are repeated from the 1st to the 5th parcel.

The project is located in the western part of the city, near the Tejo River, between two established urban areas. The housing volumes are displayed parallel to each other and perpendicular to the Avenue. They draw a 45 degrees angle with the river, establishing a strong visual relation with the landscape. This position is ruled by a correct orientation and insulation because the shopping block is opened to the pedestrian way, where public movement is, and the housing block solar exposition is correct. The arrival itinerary is done by the Avenue where can be found the commercial unities, that limit space for the vehicles circulation, and above them appears the suspended 5 blocks. It creates an expressive image of architectural kinetic movement, a conjugation of vertical volumes and a horizontal effect by the repetition series, transmitting a concept so connected to the Modern Movement – the Velocity.

In the block relationship with the ground, there is a clear reference to the 1927 Le Corbusier point o architecture – "the house assents in pilotis"; witch is assumed with big formal value. The block plan is rectangular with 11.10m width and 47.40m length (figure 1). The building has the height of 8 habitation storeys plus the pilotis height at the ground floor. The "free plan" is also a reference because the house was conceived in a way of flexibility in use. But, the 12 structural plane frames define the architectural plan of the floor type, with 6 duplex apartments. The distance between frame's axes is 3.80m. Each frame is supported by two columns and has one cantilever beam on each side with 2.80m span, resulting in 13 modules with the rhythm: A-B-B-B-B-B-B-B-B-B-B-B-A. The two A modules are associated with 2 B modules making 2 house types, the other B modules create 4 house types.

The access to the houses consists on an innovation. It as clear functional system, with 3 halls in the ground floor connected to 6 apartments each. This rational distribution access system reveals a public area economy inside the building. The service circulation is the block's "dorsal spine", an exterior gallery with 34m length, semi closed with vertical concrete slabs connecting the house's entrances with the kitchen's doors. This horizontal movement for people and goods is afterwards conducted to the ground floor or to the terrace by a staircase and a functional cargo-elevator.

Figure 1. General views of the building block under analyses

The structural plan presents an adequate solution to the architecture's objectives. "From the beginning of the studies, it was a permanently concern, to conceive a resistant structure concept, simple, elegant and economic. And it looks like the objective was well succeeded because since the beginning of the project elaboration there wasn't any need for changing the primitive structure".

The structural plan for the housing block of the Unity Type A is comprised by 12 transversal reinforced concrete frames, formed each one by two columns and 3 beams at each storey, two of them in cantilever. The structural design was initially made just for vertical loads, without considering the columns bending moment. Afterwards, new designs were developed, now considering in a simplified way the horizontal loads, corresponding to the wind, using the Cross's method in the bending moments distribution calculation. J. V. Dias does not considered in its design the seismic action. He refers "the low probability of simultaneous occurrence of wind and seismic actions in the same direction and at their maximum intensity". Dias concludes that "this building-type has superior safety conditions than the majority of Lisbon's buildings". Later, the importance of considering seismic actions in structural element's design was recognized. It was delivered a new design project according to an article of Maria Amélia Chaves and Bragão Farinha published in "Técnica" Magazine. Horizontal forces, proportional to the floor's mass were considered in the frame nodes. But, the structural analysis was only made in the transversal direction. The structural engineer concludes that wind forces induce larger demands than seismic loads, resulting in larger cross-sections. The Engineer Ramos Cruz, responsible for the construction, did a new design project for the 3rd parcel. He presented new calculations based on the primitive project, but he changed the original structural floor by a reinforced concrete slab. He advocates that with this continuous rigid slab, rigid diaphragm behaviour is guaranteed. Another modification is the introduction of reinforced concrete walls in the staircases at the ground floor, one of them was continuous in all binding's height. However, these RC elements were not detected in the technical visit to the building [6].

3 DESCRIPTION OF THE STUDIED STRUCTURE

The main objective of the work presented in this paper is to investigate the global behaviour of Modern Architecture Lisbon buildings, and to identify their weakness under seismic loadings.
The building geometry and dimensions of the RC elements and infill walls were given in the original project [1950-1956], and were confirmed in technical visits [6]. As already presented in previous sections, the building under study has nine storey and the structure is mainly composed by twelve plane frames, oriented in the transversal direction (direction Y, as represented in figure 2). The building was analysed with a simplified plane model for each direction (X - longitudinal direction, Y - transversal direction).

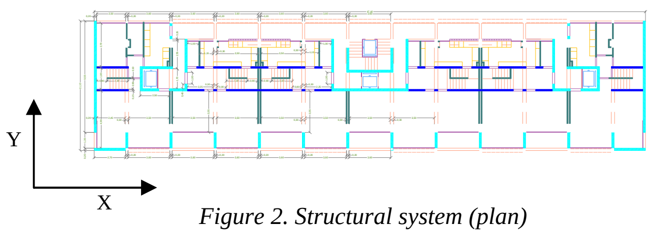


Figure 2. Structural system (plan)

The twelve transversal plane frames have the same geometric characteristics for all beams and columns. However, three different frame-types were identified, according to reinforcement detailing differences.
A peculiar structural characteristic of these type of buildings, with direct influence in the global structural behaviour, is the empty ground storey, without infill masonry walls. Furthermore, at the ground storey the columns are 5.5m height. All the upper storeys have an inter-storey height of 3.0m. Therefore, it is introduced a soft-storey mechanism at these level.
In the numerical models for the analysis of the building in the two independent directions (X and Y), it was considered a concrete slab with 1.25m width and 0.20m thick. A detailed definition of the existing infill panels, in terms of dimensions, location and materials were considered in the structural models.

For the building analysis in the transversal direction (Y), it was assumed an equivalent model defined as the association of the three frame-types, interconnected by rigid strut bars, as showed in figure 3. In this global model, the geometric and mechanical characteristics of each frame are multiplied by the number of occurrences of each frame-type.
For the analysis in the longitudinal direction (X), and because the double symmetry in plan, it was studied just one quart of the building. For the global model results a six columns structure linked at all storey levels by the RC slabs. No full-bay infill panels exist in the longitudinal direction. Therefore, an external simplified global infill masonry model was considered, as represented in figure 4, connected, at storey levels, throw rigid struts to the RC structure.

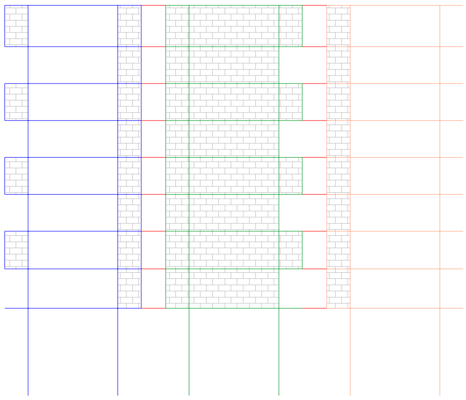


Figure 3. Structural system for the analysis in the transversal direction (Y)

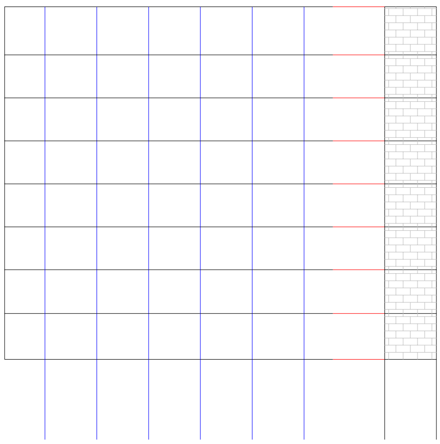


Figure 4. Structural system for the analysis in the longitudinal direction (X)

4 MODELS DESCRIPTION

Nowadays, in the analysis of structures subjected to seismic actions, the use of non-linear behaviour laws and hysteretic rules reveals a great advantage, because it makes possible a more rigorous representation of the seismic structural response.
To simulate the structural behaviour of the building presented in the previous sections, it was used a computer program PORANL, that contemplates the non-linear bending behaviour of RC elements (beams and columns) and the influence of the infill masonry panels in the global response of the buildings.

Each RC structural element is modelled by a macro-element defined by the association of three bar finite elements, two with non-linear behaviour at its extremities (plastic hinges), and a central element with linear behaviour, as represented in figure 5.

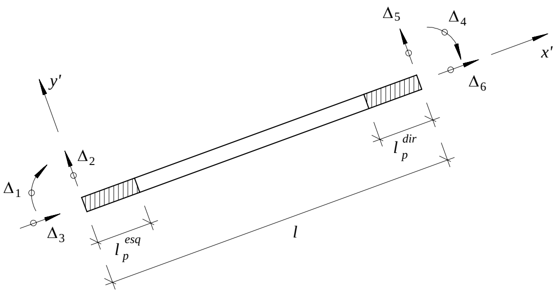


Figure 5. Frame macro-element [4]

The non-linear monotonic behaviour curve of a cross-section is characterized through a tri-linear moment-curvature relationship, corresponding respectively to: the initial non-cracked concrete, concrete cracking and steel reinforcement yielding [4]. The monotonic curve is obtained using a fibre model procedure (see figure 6), from: the geometric characteristics of the cross-sections, reinforcement and its location, and material properties.

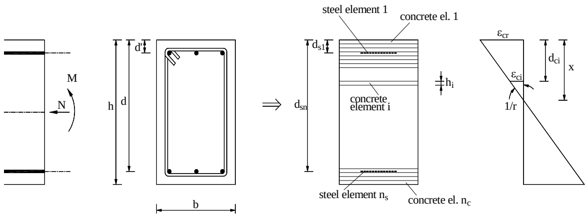


Figure 6. Fibre model for RC elements

The non-linear behaviour of the plastic hinge elements is controlled through a modified hysteretic procedure, based on the Takeda model, as illustrated in figure 7. This model developed by Costa [5] represents the response evolution of the global RC section to seismic actions and contemplates mechanical behaviour effects as stiffness and strength degradation, pinching, slipping, internal cycles, etc.

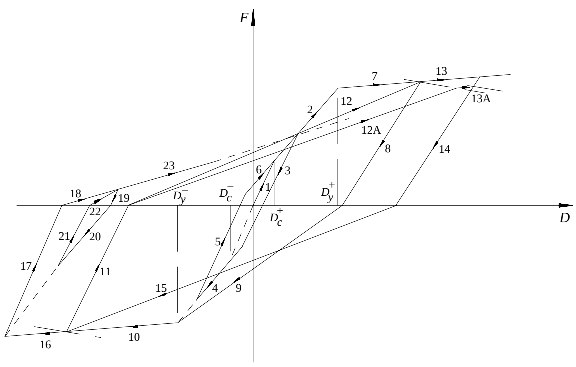


Figure 7. Hysteretic model for RC elements [4]

To represent each infill masonry panel, an improved macro-model, based on the bi-diagonal equivalent strut model, is used (figure 8). The proposed macro-model was implemented in the non-linear structural analysis program PORANL [7]. The macro-model adopted represents the non-linear behaviour of an infill masonry panel and its influence in the global RC structural behaviour under static or dynamic loading.

The monotonic behaviour curve of each panel depends on the panel dimensions, eventual openings (dimensions and position), material properties (bricks, mortar, and plaster), quality of the handwork, interface conditions between panel and the surrounding RC elements. The behaviour curves can be obtained from empirical expressions or from experimental results [8, 9].

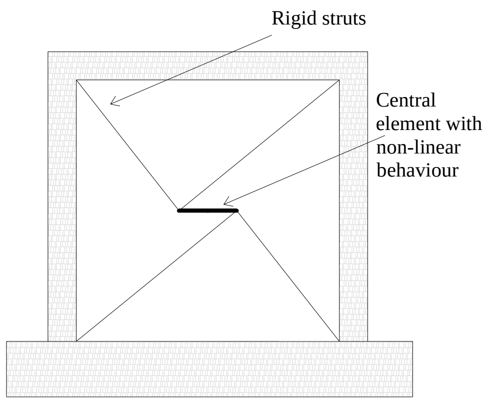


Figure 8. Infill masonry panel macro-model

The non-linear behaviour of the infill masonry panels subjected to cyclic loads is controlled through an hysteretic procedure and rules, illustrated in figure 9, and represents mechanical effects as stiffness and strength degradation, pinching, and internal cycles.

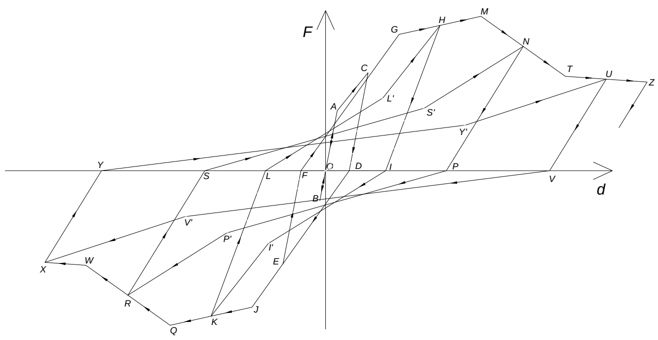


Figure 9. Hysteretic model for infill masonry panels [7]

5 STATIC LOADS, MASSES AND DAMPING

For the numerical analyses, constant vertical loads distributed on beams were considered in order to simulate the dead load associated to the self-weight of RC structural elements, infill walls, finishing's, and the correspondent quasi-permanent value of the live loads, totalising a value of 8.0kN/m^2.
The mass of the structure was assumed concentrated at storey levels. Each storey has a mass, including the self-weight of the structural and non-structural elements, infill walls and finishings, and the quasi-permanent value of the live loads, of about 4Mtons. For the dynamic analysis, the storey mass is assumed to be uniformly distributed on the floors.
For each structural model, a Rayleight damping matrix, with 1% damping ratio for the first two natural modes, was considered, according with:

$$[C] = \beta[K] + \alpha[M] \tag{1}$$

where the coefficients β and α are calculated for the damping ratio imposed for two modes of vibration (1% in these analysis). [K] and [M] are the stiffness and mass matrices of the structure, respectively.

6 NATURAL FREQUENCIES AND MODAL SHAPES

A first validation of any structural numerical model can be achieved comparing the experimentally measured and the analytically estimated natural frequencies. In table 1 are listed the four first natural frequencies computed, for the building and for each direction (X and Y).
To validate the numerical building models, in the two independent directions, it were measured the first natural structural frequency, with a seismograph and for the ambient vibration. The measured first frequency for each direction is indicated, in brackets, in table 1.

Frequencies	Direction	
	Longitudinal X (Hz)	Transversal Y (Hz)
1st	1.08 (1.17)	1.75 (1.56)
2nd	5.67	6.41
3rd	6.32	8.14
4th	8.10	8.80

Table 1: Natural frequencies for directions X and Y

A good agreement was found between the experimentally measured frequencies (1.17Hz for the longitudinal direction and 1.56Hz for the transversal direction [6]) and the frequencies estimated with the structural numerical models (1.08Hz for the longitudinal direction and 1.75Hz for the transversal direction), which constitutes the first validation of the numerical model developed. In figure 10 are represented the first natural mode shapes calculated for each direction.

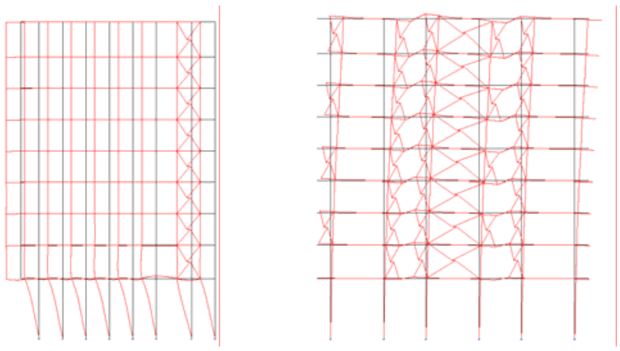


Figure 10. Natural vibration modes ($f_{1,X}$ = 1.08Hz and $f_{1,Y}$ = 1.75Hz)

From the analysis of the shapes of the first vibration modes, in both directions, it is clear that seismic actions will induce a soft-storey mechanism response. This conclusion will be confirmed with the earthquake result analysis in the forwarding sections.

7 EARTHQUAKE INPUT SIGNALS

Three artificial earthquake input series were adopted for the seismic vulnerability analysis of the building. The first series (A) was artificially generated for a medium/high seismic risk scenario in Europe [12], for various return periods (table 2). The second and third series (B and C, respectively) were generated with a finite fault model for the simulation of a probable earthquake in Lisbon [11], calibrated with real seismic actions measured in the region of Lisbon. The earthquakes of the B and C series were scaled to the peak ground acceleration of series A, for each return period. In table 2 are presented the peak ground acceleration and the corresponding return period for each earthquake's intensity.

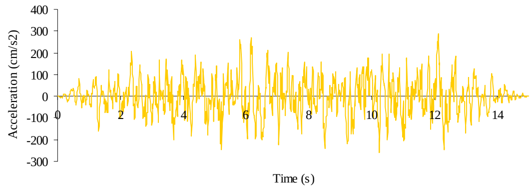


Figure 11. Accelerogram A

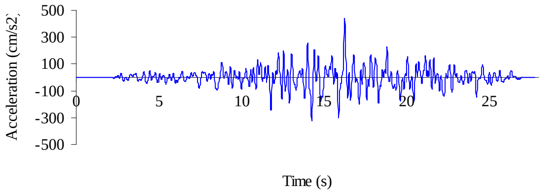


Figure 12. Accelerogram B

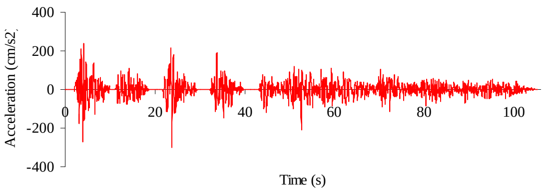


Figure 13. Accelerogram C

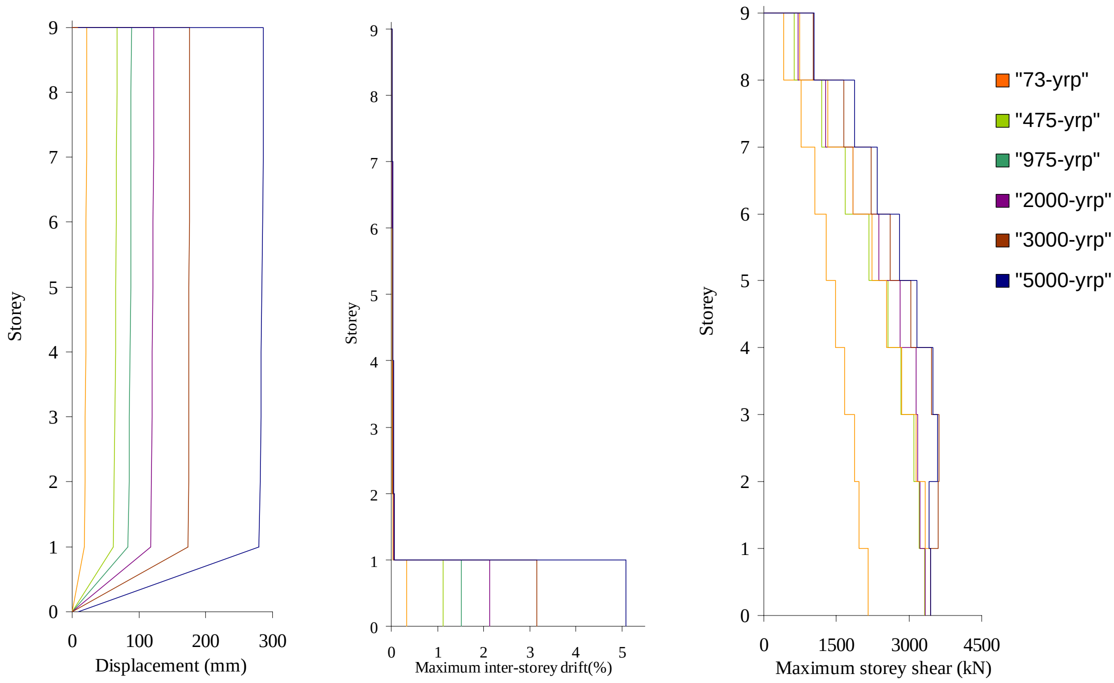
Return period (years)	Peak ground acceleration (×g)
73	0.091
475	0.222
975	0.294
2000	0.380
3000	0.435
5000	0.514

Table 2: Reference earthquake action (peak ground acceleration and corresponding return period)

8 RESULTS ANALYSIS

As observed in the free vibration shape modes, the structural response of the building, in both directions, clearly induces soft-storey mechanism behaviour (at the ground floor level). This structural behaviour leads to large storey deformation demands at the first storey, while the upper storeys remain with very low deformation levels.

In figures 14 and 15 are illustrated, for the longitudinal and transversal direction, respectively, the numerical results in terms of envelop deformed shape, maximum inter-storey drift, and maximum storey shear, for each earthquake input motion of the series A (73, 475, 975, 2000, 3000, 5000 years return period).



a) b) c)

Figure 14. Results for the longitudinal direction (X) and earthquakes of the series A: a) envelop deformed shape; b) maximum inter-storey drift profile; c) maximum storey shear profile

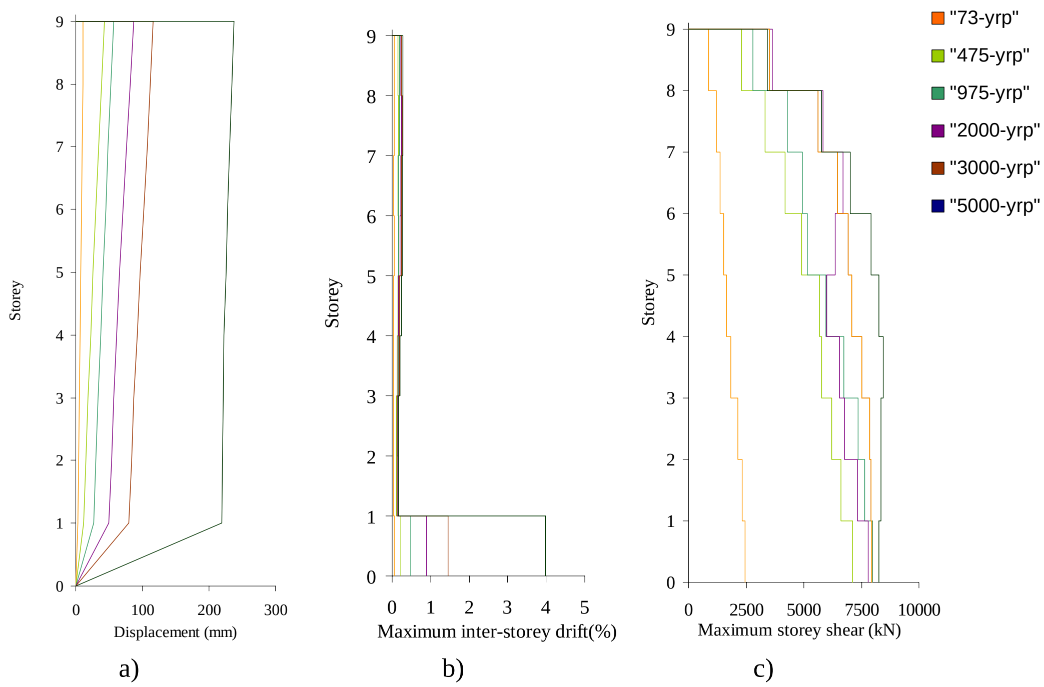


Figure 15. Results for the transversal direction (Y) and earthquakes of the series A: a) envelop deformed shape; b) maximum inter-storey drift profile; c) maximum storey shear profile

From the analysis of the results in terms of building envelop deformed shape and inter-storey drift profile, for both directions, it can be concluded that the deformation demands are concentrated at the first storey level. In fact, the absence of infill masonry walls at the ground storey and the larger storey height (5.50m for the 1st storey and 3.00m for the upper storeys), induces an important structural irregularity in elevation, in terms of stiffness and strength.

For all the structural elements (columns and beams), and for all the seismic input action levels, the shear force demand assumes a value inferior to the corresponding shear capacity, which confirms its safety in shear.

Vulnerability curves

In this section are compared, for the three earthquake series of input motions, the vulnerability curves in terms of maximum drift at ground storey, maximum 1st storey shear and maximum top displacement, for the longitudinal and transversal directions.

In figures 16 and 17 are plotted the vulnerability curves, for the longitudinal and transversal directions, in terms if the maximum 1st storey drift, obtained from the numerical analysis. Results show that, for the 1st storey, the maximum inter-storey drift demand for the longitudinal direction is larger than for the transversal, being the most vulnerable the longitudinal direction of the building.

In figures 18 and 19 are represented the vulnerability curves in terms of maximum 1st storey shear force. In figures 20 and 21 are represented the obtained vulnerability curves in terms of maximum top displacement. Shear demand at 1st storey does not increase for earthquake input actions larger than the corresponding to the return period of 475 years, inducing demands increasing just in terms of deformation, as can be observed in the results in terms of 1st storey drift and top displacement.

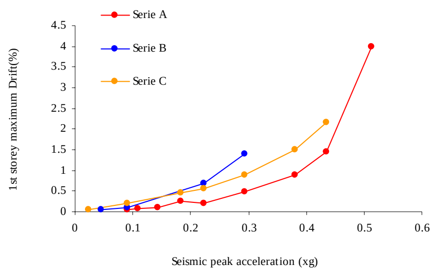


Figure 16. 1st storey maximum drift vs. peak acceleration (transversal direction - Y)

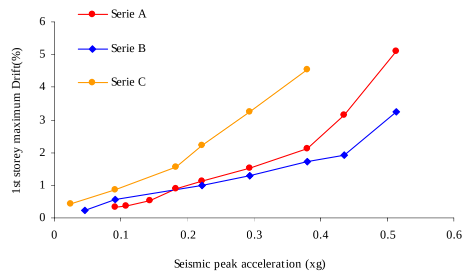


Figure 17. 1st storey maximum drift vs. peak acceleration (longitudinal direction - X)

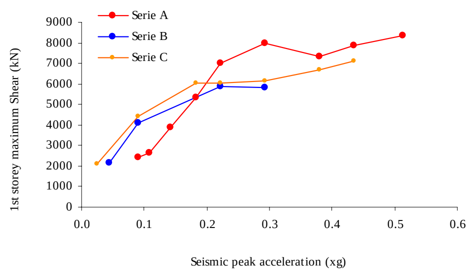


Figure 18. 1st storey maximum shear vs. peak acceleration (transversal direction - Y)

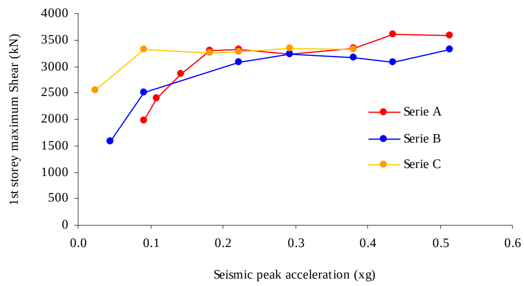


Figure 19. 1st store maximum y shear vs. peak acceleration (longitudinal direction - X)

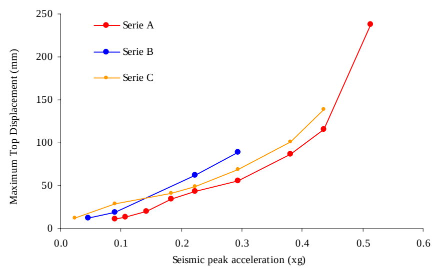


Figure 20. Maximum top displacement vs. peak acceleration (transversal direction - Y)

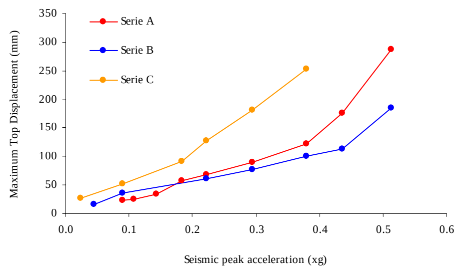


Figure 21. Maximum Top displacement vs. peak acceleration (longitudinal direction – X)

Building seismic safety assessment

As presented in previous sections, for each direction (X and Y), the building structure was analysed for three series of earthquakes, in order to estimate deformation demands, and consequently damage levels for each input earthquake intensity.

The obtained results allow verifying the safety according to the specified hazard levels, for example, the proposed in VISION-2000 [13] and ATC-40 [14] recommendations. In tables 3 and 4 are presented the acceptable inter-storey drift limits, for each structural performance level, according to the ATC-40 [14] and in VISION-2000 [13] proposals, respectively.

	Performance Level			
	Immediate Occupancy	Damage Control	Life Safety	Structural Stability
Drift Limit	1%	1-2%	2%	$0.33\frac{V_i}{P_i} \approx 7\%$

Table 3: Inter-storey drift limits according to the ATC-40 [14]

	Performance Level			
	Fully Operational	Operational	Life Safe	Near Collapse
Drift Limit	0.2%	0.5%	1.5%	2.5%

Table 4: Inter-storey drift limits according to the VISION-2000 [13]

In figures 22 and 23 are represented the vulnerability functions in terms of maximum 1st storey drift, already presented in the previous section, with indication of the safety limits proposed at the ATC-40 [14] and VISION-2000 [13] recommendations (as summarised in tables 3 and 4, respectively).

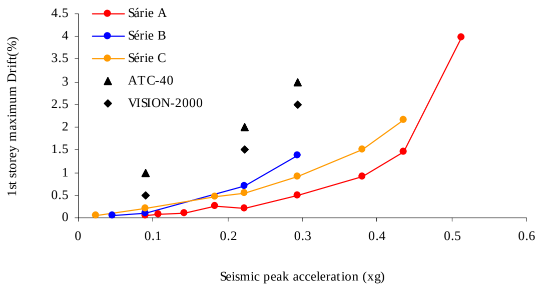


Figure 22. Maximum 1st storey drift vs. peak acceleration and safety limits (transversal direction - Y)

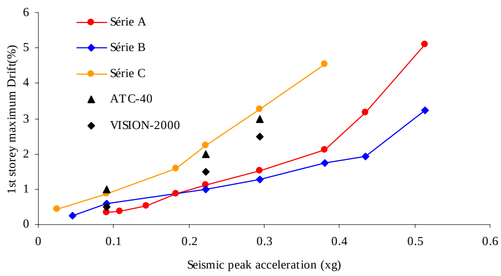


Figure 23. Maximum 1st storey drift vs. peak acceleration and safety limits (longitudinal direction - X)

Comparing the maximum storey drift demands with the safety limits proposed at the ATC-40 and VISION-2000 recommendations, it can be concluded that the building safety is guaranteed in the transversal direction (Y), for the three earthquake input series considered. For the longitudinal direction (X), the safety is guaranteed for earthquake series A and B, but not for C series.

9 PROPOSED RETROFITTING SOLUTION

For the improvement of the seismic response of the building under study it was analysed a retrofitting solution, intending to reduce the soft-storey mechanism. This solution aims to reduce the deformation demand at the ground floor level. More specifically, it was proposed a x-bracing system with a shear-link dissipation device associated (see figures 24, 25 and 26), which can increase stiffness and damping of the building, consequently reduce the deformation demands. This retrofitting solution is based on a solution studied by Varum [9].
The adoption of a x-bracing retrofitting solution was proposed due to the efficiency in reducing the deformation demands of the building, and on other hand due to this retrofitting solution do not changes significantly the architecture (only applied at the ground floor) (see figures 24 and 25).

Many alternatives for the location of the bracings can be chosen, namely in the central or external bays (see figures 25 and 26). It was developed and implemented a new numerical model, in the computational program (VisualANL) to simulate the non-linear behaviour of the device. The proposed model was implemented in the computer program and calibrated with experimental results on a full-scale cyclic test in a frame retrofitted with the same dissipative device [9]. The hysteretic behaviour and rules and the results of the calibration analysis are presented in figures 27 and 28, respectively.

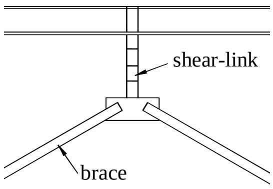


Figure 24. Shear-link (energy dissipation)

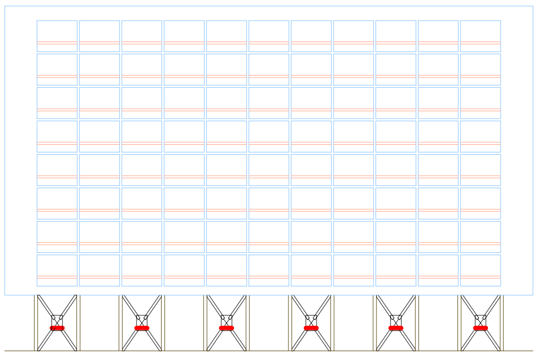


Figure 25. Location of the shear link in the longitudinal direction

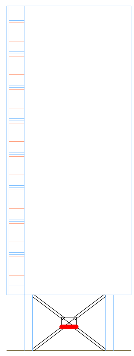


Figure 26. Location of the shear link in the transversal direction

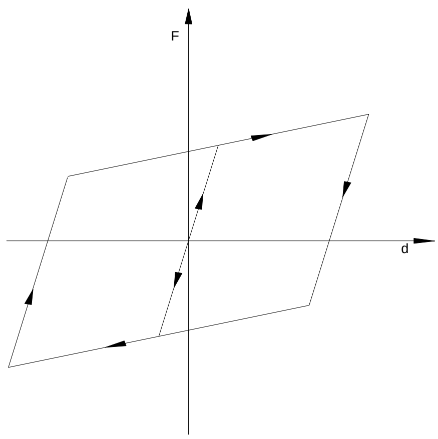


Figure 27. Hysteretic behaviour of the shear-link implemented in VisualANL

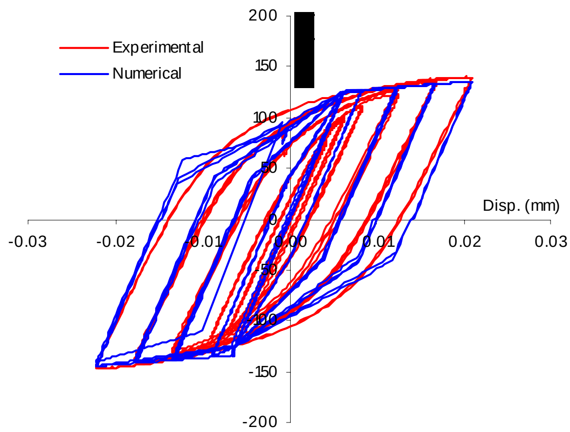


Figure 28. Calibration of the numerical model for the device cyclic behaviour

10 CONCLUDING REMARKS

The global structural safety of a modern architecture building at the Infante Santo Avenue was investigated. Although the results indicate the building safety for the Basic Objectives according to the international seismic recommendations (ATC-40 [14] and VISION-2000 [13]), it should be pointed out that additional analyses have to be performed.

The input motion earthquakes adopted for these analyses can be not fully representative of the possible seismic action in Lisbon. In other way, the level of structural damage does not dependent just of the peak ground acceleration of the earthquake. Additional analyses should be performed using other earthquake motions.

Shear capacity was verified for all the input motions. However, the model adopted for these analyses does not consider the geometric non-linearity, which can increase significantly the moments in columns and global

storey lateral deformations (drifts). Therefore, to guarantee the seismic safety verification of the building, it is judged focal to verify the results using a model that considers the geometrical non-linearities.
Finally, it was proposed a simple and economic seismic retrofitting solution which is able to reduce the seismic vulnerability associated to particular structural behaviour and deficiencies of typical existing buildings of modern architecture style. However, further analyses to validate the efficiency of the proposed retrofitting solution should be performed.

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